RATIONAL DESIGN WIND LOADS ON LATTICE TOWERS

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in Partial Fulfillment of the Requirements
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by

Capt K C Panchanathan

to the

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Feb, 1996

Dedicated

to

My Family

CERTIFICATE

It is certified that the work contained in the thesis entitled, " **RATIONAL DESIGN WIND LOADS ON LATTICE TOWERS** ", has been carried out by **Capt.K C Panchanathan** under my supervision and this work has not been submitted elsewhere for the award of a degree.

(Dr. A S R SAI)
Professor
Department of Civil Engineering
Indian Institute of Technology Kanpur

February, 1996

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Panchanathan

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SYMBOLS

A_e	Exposed area of panel to wind
C_f	Pressure/force/drag coefficient
C_m	Coefficient of virtual mass
$\bar{F}$	Mean load
F_t	Fluctuating load
G	Gust response fat or
$H(f)$	Structural frequency response function
$J_r(f)$	Joint acceptance of r^{th} mode
k_n	Generalized stiffness of n^{th} mode
M_n	Generalized mass of n^{th} mode
$R(z,z',f)$	Vertical cross correlation coefficient
$S_v(f)$, $S_F(f)$	Energy spectrum respectively of velocity and force
T	Time period over which record has been averaged
$\bar{V}$, V_t, V_1	Mean wind speed, fluctuating part of mean wind speed, mean wind speed at 10m height, respectively.
$\bar{Y}$	Mean displacement
α	Power law index of variation
Φ	Solidity ratio of panels
ϕ_r	r^{th} mode shape
ζ_n	Damping coefficient in n^{th} mode
ω	Angular frequency of vibration

ω_d	Damped frequency of vibration
σ	Standard deviation or root mean square (rms) value.
$\lvert\lambda_m(f)\rvert^2$	Mechanical admittance
f_n	Natural frequency of vibration of structure
z	Distance along the vertical direction

ABSTRACT

With the *opening up of the Indian economy*, vast developments have taken place in the area of telecommunications.The cellular telephone technology has changed the lives, making it possible for us to communicate with each other at the press of a button. As structural engineers we have to produce cost effective infrastructure to backup this fast pace of development. The intensity of loads to which a structure is subjected to has a significant impact on the overall design and cost. Wind force is a predominant force on lattice towers and often governs the structural design. The guidelines for evaluating wind loads are incomplete and inconsistent in approach. The ambiguities, lacuna and shortcomings, if any, of some of the codes of practice and their implications on the overall design are highlighted with the help of examples. The analysis of the tower has been done by the stiffness method of structural analysis. A pre-processor package has been developed that generates the input for further analysis by the **SAP 80** package. Tubular sections, as mentioned in IS: 1161-1979 have been used.

CHAPTER I

INTRODUCTION

1.1 General

History has shown that the wealth and prosperity of nations has been primarily due to the strides made by them in the fields of Science and Technology. Communication is an area that has made Conquest of Space to marriages in Cyberspace possible. Today with the the introduction of cellular telephone technology in the world, a person can be reached anywhere and at any time. Microwave and cellular telephone towers are essentially similar, but for the height and the operating range. A brief look at the functional requirements of both these towers is presented in Section 1.2.

1.2 Specification of Towers

1.2.1 Microwave Towers

Microwave towers are self-supporting structures with an average height of 100-150 m. They are generally divided into 15-20 panels, each of 6-10 m in height. The top 15 m is generally vertical, in order to facilitate the erection of antennas. Panels up to a height of 60-70 m from the ground are '**K**' braced while the rest of the panels may be either '**X**' or '**XB**' braced. The towers may be triangular or square in plan, depending on the height and the structural members used. Rest

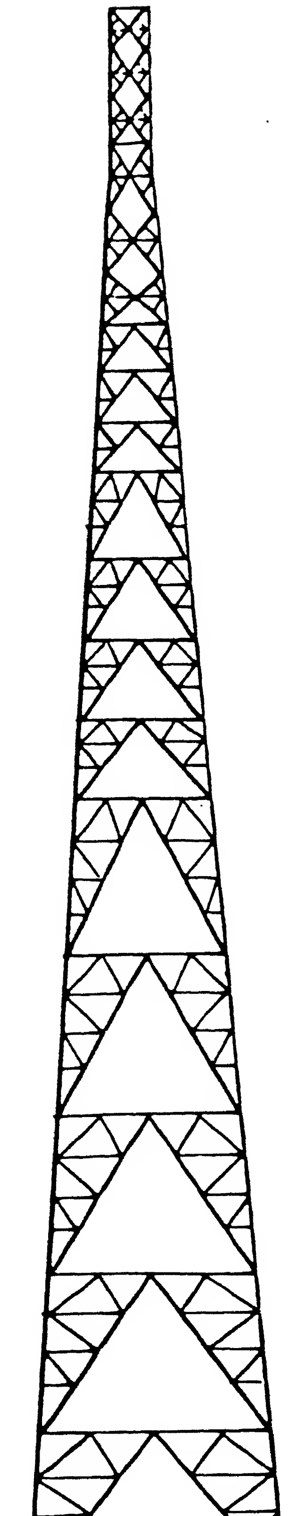
platforms are provided at every 30 m interval from the bottom. Taller towers are also provided with elevators in addition to ladders. Depending on the requirement three or four antennae of 3.5 m diameter may be mounted at the top or even at lower heights.

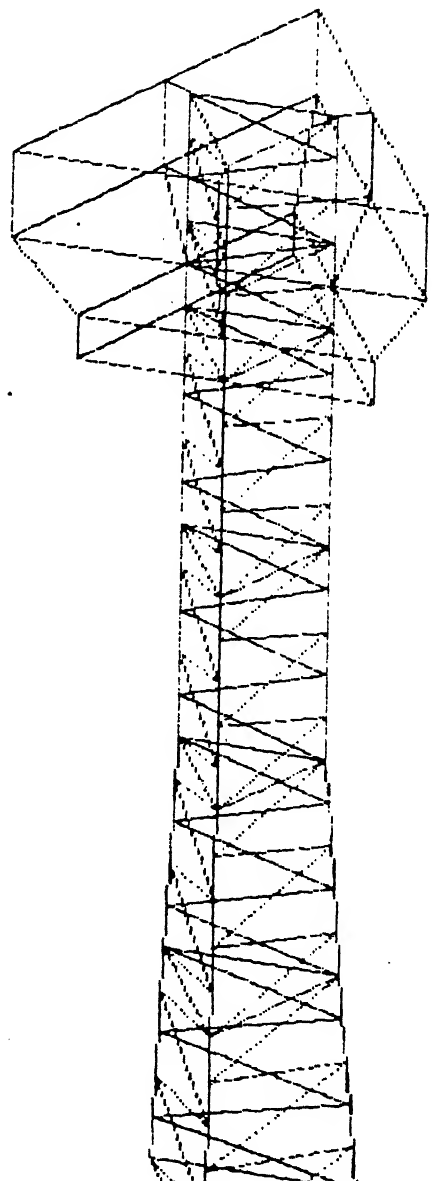
1.2.2 Cellular Telephone Towers

Cellular telephone towers are light weight self supporting structures with an average height of 20-30 m. Each tower services a cell, with a range of operation limited to about 5 km. Mounted on top of the tower are 6-9 antenna arrays weighing about 10 to 15 kg each. Arrays are of two types, transmitting and receiving, with a minimum separation of 5 m between them. Fig 1.1 shows the configuration of antenna at the top. As these towers are light in weight they are mounted on top of buildings, for which the existing building slabs may have to be strengthened. The foundation of the tower must be designed taking into account the ground conditions.

1.3 Review of Literature

A lot of work has been done in various methods of structural optimization from the surveys of Wasiutynski and Brandt[1] in 1963 and Sheu and Prager[2] in 1968. Geometric changes in optimization process to reduce weight was studied by Kumar and Chan[3]. Later Kumarasamy[4] has applied configuration optimization to transmission line towers. An effort to study the in-plan





configuration optimization of microwave towers was carried by Roy[5] in 1993. Earlier, design of tower structure was carried out by idealizing it into a two dimensional problem and analyzing the determinate structure after deleting the redundant members. Now, with the advent of digital computers, it is possible to analyze the towers as a three dimensional statically indeterminate structure subject to many load conditions. Natarajan[6] has presented a simpler version of the frontal solution technique suitable for structural analysis with a large number of members. His method is ideally suited for small and medium height towers. Rangaswami and Jayaraman[7] have carried out the stiffness method of structural analysis for a transmission line tower considering it as a two dimensional partially rigid jointed framed structure.

It is now a well established fact that in the design of almost all tall structures, ranging from skyscrapers to radio telescopes, dynamic wind pressure is a major consideration for the structural design. Traditionally these pressures were calculated on the assumption that the fluctuations in the velocity owing to gusts could safely be disregarded and that velocity could be taken as invariant with time and space. Sherlock[8], a pioneer in this field, advocated the use of an average instead of a maximum gust velocity together with certain 'gust factors' which would cater to the additional effects of gusts. Davenport[9], based on the work of Sherlock, modified the foregoing approach to include the history of loading pattern and effects of roughness of terrain and applied the statistical concepts of the stationary time series to evaluate the response

of structures to turbulent gusty winds. He formulated an empirical expression for the spectrum of horizontal gustiness in high winds. Simiu[10] in 1973, compared the works of Davenport, with that of Vellozi and Cohen[11] regarding the alongwind cross correlation of the fluctuating pressure and of its effect upon the magnitude of response of a structure. He concluded that while Davenport over estimated the response, Vellozi and Cohen under estimated the response. Later in 1975, Simiu[12] then proposed a wind spectra for alongwind response, applicable for both the high and low frequency gusts, and showed how the dynamic alongwind response of tall, flexible, lightly damped structures are significantly smaller using this spectra rather than the spectra proposed by Davenport.

Recently, the Codes of Practice of countries like the United States of America[13], United Kingdom[14] and India[15] have recommended simple expressions for computing the alongwind and crosswind response of structures based upon the works of the authors mentioned in references 8 to 12. In 1983, Jun Kanda[16] made a study of the reliability of the gust response prediction considering the height dependant turbulence parameters. Solari[17] presented a new approximate analytical method of computing the alongwind response as an alternative to the classical use of graphs and tables.

1.4 Scope of Present Work.

A parametric study on the effect of variation of structural parameters on the overall design of tower has been carried out. The analysis has been done on SAP 80[18](structural analysis programs for static and dynamic analysis of structures) package, for various towers having different H/D (overall height to diameter of inscribing circle) ratios and in-plan configurations respectively. To considerably simplify the analysis, a general purpose computer program to generate the input in required format, was prepared in FORTRAN. The details of which are covered separately . Both static and dynamic analyses has been carried out. Earthquake analysis has also been done as per the provisions of IS :1893-1984[19], Criteria for earthquake resistant design of structures. In Chapter 2, a comparison of the wind loading provisions currently being practiced in India, UK, and USA, and their effect on the design of a tower has been made. Chapter 3 deals with the dynamic alongwind response of a structure, wherein the underlying concepts behind evaluation of gust response factors are studied comparing the works of pioneers in this field (Davenport, Simiu, etc.,). Chapter 4 has been dedicated to the results of the parametric study carried out on towers of different H/D ratios, making use of the provisions discussed in Chapters 2 and 3. A critical look at the current practice being followed in India has been discussed at the end of Chapter 5.

CHAPTER II

COMPARISON OF CODAL PROVISIONS

2.1 General

In this chapter a comparison of the wind loading provisions given in ASCE: 7-93, Minimum design loads for Buildings and Structures, BS: 8100-1986, Standard for Loading of Lattice Towers and Masts, and the IS: 875-1987, Code of Practice for Design Loads (other than earthquake) for Buildings and Structures, Part 3 Wind Loads is made. The study has been made primarily to understand the rationale behind codification in different countries and to select an optimum value for design.

2.2 Wind Characteristics.

If the air flow does not encounter any obstacles, it moves under the action of pressure gradients with a velocity known as the gradient velocity. Near the ground the wind is retarded due to friction with the irregular and rough terrain. The effect of this friction upon the flow is considerably less as the height above the ground increases. As per IS: 875, the value of gradient height has been specified to be 250, 300, 400 and 500 m respectively for Terrain Category I, II, III, and IV. Due to this friction offered by these obstructions the wind flow becomes turbulent leading to random variations in the wind velocity, $V(z,t)$ with height. Thus, $V(z,t)$ is decomposed into the mean value $\bar{V}(z)$, and the fluctuating part $v(z,t)$. In order to

understand the rationale behind the parameters considered, some relevant definitions are presented in the following section.

2.2.1 Definitions

(a) **Reference wind speed:** This is the basic wind speed data applicable at a height of 10 m above ground level in flat open terrain. The averaging time may vary, depending on the code of practice.

(b) **Design wind speed:** It is the modified wind speed to include the effects of local topography, terrain roughness, structure size, risk coefficient, and most importantly, the variation of wind speed with height.

(c) **Solidity Ratio (ϕ):** Solidity ratio is defined as the ratio of the projected area of all individual members normal to the wind flow direction, to the projected area of the structure normal to the direction of flow

(d) **Pressure Coefficient (C_f):** When wind attempts to negotiate around a body, a considerable amount of turbulence is created near the surface of the body. This turbulence creates fluctuating pressures on the surface. The non-dimensional ratios of wind induced pressure on a surface, to the dynamic free field wind pressure are termed as pressure coefficients. They are dependant on the solidity ratios, type of structural members being used, orientation of the structure to the direction of wind flow, and in some cases to the Reynold number (R_e) of flow.

2.3 Comparison of the Wind loading Provisions.

The wind induced pressure on a structure is a function of wind properties as well as shape and size of the structure. Design wind

pressure 'P' acting normal to a surface as per IS: 875, is expressed as

$$P(z) = 1/2\ \rho\ (K_1\ K_2\ K_3\ V_b)^2 \qquad (2.1)$$

where,

$P(z)$ = Design wind pressure (Pa)

ρ = Air mass density (kg / m^3)

V_b = Basic wind speed (m / s)

K_1 = Risk coefficient factor,

K_2 = Terrain, height, structure size factor, and

K_3 = Topography factor.

The load determination procedure of the other two Codes is essentially based on Eq (2.1). Approaches in each standard are different owing to the difference in connotations of the terms being used by that standard. The comparison of the terms used in the foregoing equation and their equivalence in other Codes is as shown in Table 2.1 .

Table 2.1 Comparison of Terms

Terms in Eq 2.1 (as per IS 875)	**Equivalent Terms as per**	
	ASCE 7 93	**BS 8100 1986**
V_b Reference wind speed at 10 m level, an open terrain, and with a return period of 50 years.	V Fastest mile speed in mph, at 33 ft in in open terrain , with a return period of 50 years.	V_B Max mean hourly speed in m/s, a 10 m level with a return period of 50 years.
K_1 Risk Coefficient factor	I Importance Factor	γ_v Partial safety factor.
K_2 Terrain, height, size factor	K_z Velocity pressure exposure factor.	K_R Terrain roughness factor *
K_3 Topography factor	None	K_μ Terrain profile coefficient.
None	None	K_D Wind direction factor

A popular way of describing the variation of mean wind velocity with height within the atmospheric boundary layer has been through the power law given by Davenport ,

$$V\ (z) = V_{ref} (z\ /\ z_{ref})^{\alpha} \qquad (\ 2.2\ )$$

where, α is the power law index of variation, and V_{ref} is the mean wind velocity at z_{ref}, 10 m. Conventional values of α, are taken as 1/10, 1/7, 1/4.5, 1/3 for Categories I to IV respectively. Alternatively, Ekman had postulated the logarithmic profile of variation of wind velocity with height ie.

$$V\ (z) = \frac{1}{\kappa}\, u_*\ \ln \left[\frac{z - d}{z_o} \right] \qquad (\ 2.3\ )$$

where, κ is Von - Karman's constant equal to 0.4, u_* is the shear velocity given by $\sqrt{\tau_o/\rho}$ where τ_o is the shear stress on the terrain surface and ρ is the mass density of air. In the foregoing expression 'd' represents the zero plane height, where the wind velocity is considered zero. In IS: 875, 'd' has been specified as equal to zero for Categories I, II, III, and for Category IV the value has been given as 10 m. Further z_0 is the roughness length and the values specified are 0.002, 0.02, 0.2 and 2.0 m respectively for Categories I to IV.

The code has adopted the logarithmic profile of variation of wind velocity with height, and this is incorporated in the $\bar{K}_2$ or hourly mean wind speed factor. In BS: 8100, the variation of wind speed with height has been expressed by the following relation:

$$Vz = Vr\ (\ z - he\ /\ 10)^{\alpha} \qquad z \geq 10 + he \qquad (\ 2.4\ )$$

$$= Vr/2\ (\ 1 + z\ /(10 + he))^{\alpha} \qquad z < 10 + he \qquad (\ 2.5\ )$$

Vr = site reference wind speed

α = power law index of variation of speed with respect to height, for flat and open terrain, α = 0.165

he = effective height of surface obstruction, 0.0 for flat open terrain.

ASCE: 7-93, follows the power law of variation of velocity with height, as is evident from the K_z (velocity pressure exposure coefficient) factor given. The code specifies the following

relationship of K_z with height:

$$K_z = 2.58 \left(\frac{z}{z_g}\right)^{2/\alpha} \quad \text{for} \quad 15 \text{ feet } (4.5 \text{ m}) \le z \le z_g \qquad (2.6)$$

$$K_z = 2.58 \left(\frac{15}{z_g}\right)^{2/\alpha} \quad \text{for} \quad z < 15 \text{ feet } (4.5 \text{ m})$$

2.3.1 Reference Wind Speeds.

An important parameter in defining the reference wind speed is the averaging time for mean wind speed. In the American standard the reference wind speed is the fastest mile wind speed. The British standard specifies the mean hourly wind speed at 10 m height and having an annual probability of occurrence of 0.02. The Indian standard specifies a wind speed based on peak velocity averaged over 3 seconds at 10 m high level in open flat terrain having an annual probability of occurrence of 0.02 (50 year return period). These differences in the reference wind speeds and their averaging time have a dramatic effect on the dynamic wind pressures. Simiu[20] has given a relation between mean hourly wind speed and the wind speed averaged over 't' seconds, corresponding to flat and open terrain, and at reference height of 10 m (33 ft) as

$$V_t(z) = V_{3600}(z) \quad 1 + \left[\frac{0.9\ c(t)}{\ln(z/z_0)}\right] \qquad (2.7)$$

$z = 10$ m

$c(t)$ = coefficient that depends on t, see Table 2.2

$V_t(z)$ = wind speed averaged over t seconds

$V_{3600}(z)$ = mean hourly wind speed

$t = 3600/ V_f$

V_f = fastest mile wind speed

Table 2.2 , Value of Coefficient , c(t)
(Taken from Wind effect on structures, Simiu & Scanlan, 1978)

t(secs)	1	10	20	30	50	100	200	300	600	1000	3600
c(t)	3	2.32	2.0	1.73	1.35	1.02	0.70	0.54	0.36	0.16	0.00

Therefore for V_f = 100 mph., the averaging time is 36 seconds. Corresponding to a value of 36 seconds, the value of c(t) obtained from the table is about 1.615. The corresponding mean hourly wind speed is 77 mph (34 m/s). Table 2:3, gives the comparison of reference wind speeds.

Table 2.3, Comparison of Reference Wind Speeds

Reference wind speed	ASCE:7-93	BS:8100-1986	IS:875-1987
Averaging Time	Fastest mile	Mean hourly speed	3 second gust
Equivalent Ref wind speed to fastest mile speed of 100 mph.	100 mph (44.5 m/s)	77 mph (34 m/s)	114 mph (51 m/s)

In all the comparisons being made hereafter, basic wind speed of 50 m/s as per IS :875, which translates as 33 m/s mean hourly

windspeed, and 96.43 mph in terms of the fastest mile speed have been used

2.3.2 Design Wind Pressure

The Indian Standard defines the design wind pressure in relation with the design wind velocity as

$$p(z) = 0.6\ V^2(z) \tag{2.8}$$

$p(z)$ = Design wind pressure in Pa, at height of z m

$V(z)$ = Design wind velocity in m/s, at height z

The coefficient 0.6 (in SI units) depends upon the temperature and pressure of air and is applicable to Indian atmospheric conditions

Whereas in the American Standard, the design wind pressure is converted into velocity pressure, $q(z)$, in pounds per square feet, by use of the following relationship

$$q(z) = 0.00256\ K_z\ (I\ V)^2 \tag{2.9}$$

V = Design fastest mile wind speed

I = Importance factor, as defined earlier

K_z = Velocity pressure exposure coefficient

The British standard, has defined the wind pressure similar to the Indian standard, with the exception that the mean wind speed referred to is the mean hourly wind speed.

2.3.3 Force (Pressure/Drag) Coefficients.(C_f)

While it is understood that the variation in averaging time, has a major effect in the velocity and pressure calculations, one would expect the values of force coefficients, which are essentially a function of the solidity ratios and the orientation of the structure, only to vary marginally, depending upon the methodology of the tests carried out on scaled models in wind tunnels. Fig 2.1 shows the variation of solidity ratio along the height of a 42 m tower,which is being used as an example for comparison in the following sub-sections.

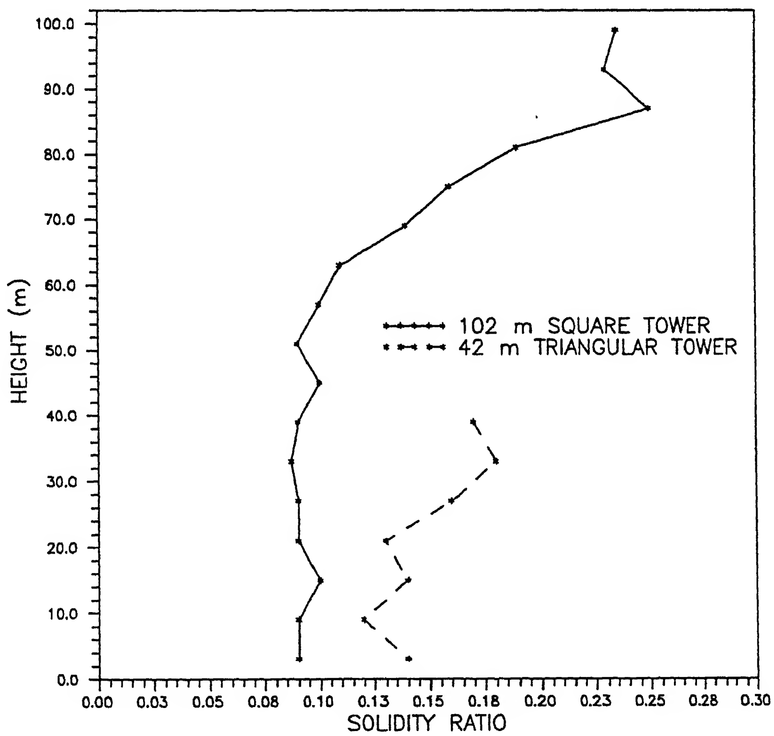


Fig 2.1 Variation of solidity ratio (ϕ) with respect to height (H), for 42/102 m towers

2.3.3.1 IS: 875

The Indian standard has specified the values for overall force coefficients (here after referred to as **FC**) for both square and equilateral triangular towers, made from angle/flat sections and rounded sections separately. For towers composed of circular sections, **FC** for both normal and diagonal wind conditions have been given separately for different flow regimes (sub-critical, and super-critical). The classification of flow regimes as sub-critical, and super-critical, have been made on the basis of Reynolds number. For tower appurtenances like ladders, conduits, wave guides, etc., a mention has been made of adopting appropriate values for **FC** but no specific values have been recommended.

2.3.3.2 ASCE: 7-93

The values of **FC** pertaining to trussed towers have been obtained from wind tunnel tests conducted in relatively smooth flow regimes. Their validity in turbulent regimes is yet to be ascertained. The values of the **FC** for trussed towers , composed of structural angles or similar flat sided members are shown in Table 2.4

Table 2.4 , Force coefficients for Trussed Towers, C_f
(Taken from ASCE 7-93)

Solidity ratio (ϕ)	Force Coefficient	
	Square towers	Triangular towers
< 0.025	4.0	3.6
0.025 to 0.44	4.1 - 5.2 ϕ	3.7 - 4.5 ϕ
0.45 to 0.69	1.8	1.7
0.7 to 1.0	1.3 + 0.7 ϕ	1.0 + ϕ

Note to Table 2.4:

1. For towers with rounded members, the design wind force shall be determined using the values in the table multiplied by the following factors:

$\phi \leq 0.29$, factor $= 0.67$

$0.3 \leq \phi \leq 0.79$, factor $= 0.67\ \phi + 0.47$

$0.8 \leq \phi \leq 1.0$, factor $= 1.0$

The standard has assumed smooth flow regimes and has not mentioned values for turbulent flow regimes. Like the IS code, use of appropriate values for FC, for tower appurtenances has been mentioned.

2.3.3.3 BS: 8100-1986

The British standard has specified both formulae and graphs for calculation of overall drag coefficients, for both the type of towers composed of angle, rounded, or a combination of both (hybrid). Separate formulas, for different flow regimes have also been included. Relevant extracts of the expressions for circular sections for both the flow regimes are as follows:

$$C_{NC} = C_1\ (1 - C_2\ \phi) + (C_1 + 0.875)\ \phi^2 \qquad (2.10)$$

$$C_{NC}' = 1.9 - \sqrt{(1-\phi)\ (2.8 - 1.14\ C_1 + \phi)}$$

$C_1 = 2.25$ or 1.9 (square/triangular towers)

$C_2 = 1.5$ or 1.4 (do)

C_{NC} = Drag coefficient, sub critical circular members.

C_{NC}' = Drag coefficient, super critical circular members.

The standard has given detailed classification of the types of tower appurtenances and the formulas to calculate the drag coefficients

2.3.3.4 Comparison of Values

Table 2.5, gives a comparison of the values of overall **FC**, obtained using the relevant expressions given in the above codes, for a 42 m triangular tower and a 102 m square tower, both composed of circular sections as show in Fig 1.1 .

Table 2.5 , Comparison of Overall Force Coefficients (C_f)

Height (m)	**Average Solidity Ratio (ϕ)**	**Force Coefficient (C_f)**		
		IS:875	**BS:8100**	**ASCE:7-93**
42	0.149	1.650	1.550	2.029
102	0.138	2.018	1.856	2.070

The values for the solidity ratio for the full tower have been arrived at by averaging the values of solidity ratios of the individual panels. It is pertinent to point out that while arriving at the value of overall **FC** for the tower, a tacit assumption that the members that compose the tower, are in the same flow regime, has been made. This has been done, because calculation of Reynolds number for each member is not feasible. While it is appreciated that the values of the **FC** have been

obtained from wind tunnel tests carried out on scaled models, their applicability to real life situations wherein the towers are subjected to fluctuating mean wind speeds over the height is debatable. Hence, a study of the variation of the **FC** along the height of a 42 and 102 m tower was made, and the results are as shown in Tables 2.6 and 2.7, respectively.

Table 2.6 , Comparison of Panel wise Force Coefficients(C_f) for 42 m, Triangular Tower

Panel No (from top)	Solidity Ratio (ϕ)	Force Coefficient (C_f)		
		IS:875	**BS:8100**	**ASCE:7-93**
1	0.174	1.605	1.778	1.955
2	0.182	1.597	1.761	1.929
3	0.157	1.621	1.813	2.005
4	0.128	1.655	1.879	2.092
5	0.140	1.641	1.853	2.058
6	0.121	1.665	1.898	2.116
7	0.140	1.643	1.858	2.064

Table 2.7 , Comparison of Panel wise Force Coefficients(C_f) for 102 m, Square Tower

Panel No (from top)	Solidity Ratio (ϕ)	Force Coefficient (C_f)		
		IS:875	BS:8100	ASCE:7-93
1	0.235	1.793	1.666	1.769
2	0.235	1.793	1.666	1.769
3	0.249	1.769	1.645	1.728
4	0.190	1.883	1.745	1.904
5	0.159	1.961	1.811	2.001
6	0.138	2.018	1.856	2.070
7	0.115	2.099	1.913	2.134
8	0.103	2.146	1.944	2.170
9	0.093	2.186	1.968	2.198
10	0.099	2.158	1.952	2.179
11	0.093	2.186	1.968	2.198
12	0.087	2.214	1.984	2.216
13	0.094	2.182	1.966	2.195
14	0.090	2.202	1.978	2.208
15	0.097	2.168	1.958	2.186
16	0.094	2.182	1.966	2.195
17	0.091	2.197	1.975	2.205

Table 2.8, shows the effect of considering the overall **FC,** vis a vis, the **FC** per panel, on the mean wind load per panel, for a 102 m tower, for the values of **FC** obtained as per IS: 875 .

Table 2.8 , Comparison of Wind Load per Panel for 102 m, Square Tower

Panel No (from top)	Solidity Ratio (ϕ)	Wind Load per Panel (kN)	
		(1)	(2)
1	0.235	12.05	13.56
2	0.235	11.89	13.38
3	0.249	15.11	17.24
4	0.190	16.70	17.90
5	0.159	18.17	18.70
6	0.138	19.32	19.32
7	0.115	19.50	18.75
8	0.103	20.15	18.95
9	0.093	20.68	19.10
10	0.099	23.77	22.23
11	0.093	24.23	22.37
12	0.087	24.47	22.30
13	0.094	27.18	25.14
14	0.090	26.77	24.53
15	0.097	28.77	26.78
16	0.094	26.65	24.65
17	0.091	21 16	19.44

(1): Load considering **FC** for each panel.

(2): Load considering overall **FC** for tower.

From the foregoing it is seen that the wind load in the bottom 10 panels are 10-15% lesser while considering an overall value for the **FC**. The percentage difference however depends upon the height of tower and other parameters. It will suffice to state that, an overall **FC** value for any tower will yield lower values of the static wind loading per panel.

2.4 Wind Load Evaluation

The mean wind load (only considering the mean wind speed and not including the effects of gusts), **W**, can be expressed as follows:

$$W = C_f A_e P(z) \qquad (2.12)$$

C_f = Force coefficient, as defined above

A_e = Exposed solid area normal to the wind direction

$P(z)$ = Wind pressure as defined in Eq

A comparison of the wind load per panel, for a 42 m and 102 m tower described earlier, is shown in Tables 2.9 and 2.10 respectively .

Table 2.9 , Comparison of Wind Load per Panel for 42 m, Triangular Tower

Panel No (from top)	Solidity Ratio (ϕ)	Wind Load per Panel(kN)		
		IS:875	BS:8100	ASCE:7-93
1	0.174	5.63	4.171	5.95
2	0.182	7.06	5.08	7.29
3	0.157	8.25	5.85	8.55
4	0.128	8.36	5.82	8.68
5	0.140	10.29	6.75	10.30
6	0.121	9.52	5.21	9.39
7	0.140	9.45	3.64	9.69

Table 2.10 , Comparison of Wind Load per Panel for 102 m, Square Tower

Panel No (from top)	Solidity Ratio (ϕ)	Wind Load per Panel (kN)		
		IS:875	BS:8100	ASCE:7-93
1	0.235	12.05	8.64	11.43
2	0.235	11.89	8.46	11.23
3	0.249	15.11	10.65	14.02
4	0.190	16.70	11.61	15.94
5	0.159	18.17	12.43	17.34
6	0.138	19.32	13.14	18.51
7	0.115	19.50	12.81	18.20
8	0.103	20.15	12.94	18.50
9	0.093	20.68	12.99	18.69
10	0.099	23.77	14.73	21.33
11	0.093	24.23	14.58	21.31
12	0.087	24.47	14.31	21.12
13	0.094	27.18	15.51	23.19
14	0.090	26.77	14.76	22.45
15	0.097	28.77	15.15	23.66
16	0.094	26.65	11.56	21.60
17	0.091	21 16	9.89	17.72

From the above tables, it is clear that there is a close similarity in the values of the wind load per panel between the IS code and the ASCE standard, while the values as per the British standard differ from the other two codes by quite a margin. This is because of the fundamental difference in the averaging times of the reference wind speeds. Also in the lower two panels, in case of both the towers, the values of the wind load as per the British standard are nearly one third the values

of the other two standards. This can be explained as BS: 8100 specifies that the variation of velocity below the reference height of 10 m is linear, whereas in the other two standards the velocity below 10 m, is taken as being equal to the reference velocity.

CHAPTER III

DYNAMIC ALONGWIND RESPONSE OF LATTICE TOWERS

3.1 General

The total response of a tower may be viewed as a sum of two parts, the mean deflection induced by the mean wind and the fluctuating deflection induced by the gusty wind. The maximum alongwind deflection of a structure at an elevation z m , from the ground may be written as

$$X_{max}(z) = x(z) + \bar{x}(z) \qquad (3.1)$$

where $\bar{x}(z)$ is the mean deflection, and $x(z)$ is the maximum fluctuating deflection in the direction of mean wind. Wind speeds used in current design specifications are usually based on either the *fastest mile wind speeds* or the *mean hourly wind speeds.* The current practice is to multiply the mean wind speed by a gust factor to allow for the fluctuations in the wind speed. Although such allowances for gusts have resulted in safe and economical designs, the neglect of both the dynamic properties and size of the structure, could result in unsafe or costly over design.

The structural loads produced by gusts depend on structural properties such as size, natural frequency and damping. The load decreases for a particular gust as these parameters increase. The gust load also increases as the surface roughness and

obstructions increase. The *gust response factor* is a measure of the effective dynamic load produced by gusts, and is intended to translate the dynamic response phenomenon produced by gust loading into simpler static design criteria. Calculation involves, a power spectrum analysis of the dynamic structural response of a linear single degree of freedom system(SDOF) with viscous damping, and uses reported measurement of wind gustiness spectra and gust correlation coefficients. The records of actual measurements carried out by Davenport(1967) indicate that the largest dynamic response generally occurs in the lowest mode or modes of vibration of the structure with higher frequencies relatively quiescent. This is principally due to the fact that the greatest energy in the wind almost invariably exists at the lower frequencies.

3.2 Structure of Gusty Wind

From the principles of fluid dynamics it is known that the general expression for the force induced on a structure due to the unsteady flow is

$$F(t) = A\left[0.5\,\rho\,C_d\,V^2(t) + C_m\,\rho\,(A_o/D)\,(dV(t)/dt)\right] \qquad (3.2)$$

where $F(t)$ is the force at any instant of time t, due to the velocity $V(t)$; A is the area over which this force acts, ρ is the density of the fluid, C_m and C_d are the coefficient of virtual mass and drag respectively. C_m relates to the effects linked to the fluid acceleration, and for the present can be neglected. Consider a SDOF system, expressed by the equation of motion

$$m\dot{y} + c\dot{y} + ky = F(t) \quad (3.3)$$

Where m, c, and k are the mass, damping and stiffness respectively of the system considered. For fluctuating input $V(t)$ the solution of the above equation is the displacement $y(t)$. Since input is a random process and the wind velocity can be distributed according to the Gaussian law

$$V(t) = \overline{V} + V_t \quad (3.4)$$

where $\overline{V}$ is the mean value and V_t is the fluctuating part of the velocity.

$$V = \frac{1}{T} \int_0^T V(t)\, dt \quad (3.5)$$

$$\sigma_v = \left[\int S_v(f)\, df \right]^{1/2} \quad (3.6)$$

where σ_v is the standard deviation and $S_v(f)$ is the *energy spectrum of velocity.* Since the mean and the standard deviation are known its probability distribution is defined. Similarity one can can express the wind load $F(t)$ as

$$F(t) = \bar{F} + F_t \quad (3.7)$$

$$\bar{F} = \frac{1}{T} \int_0^T F(t)\, dt = 0.5\, \rho\, C_d\, A\, (\bar{V})^2 \left[1 + \frac{V_t^{\,2}}{(\bar{V})^2} \right] \quad (3.8)$$

Where, $\bar{F}$ is the mean load and F_t is the fluctuating part of load Neglecting the $(V_t / \bar{V})^2$ terms, as they are very small in high winds one can now rewrite the above equations as

$$\bar{F} = 0.5\ \rho\ Cd\ A\ (\bar{V})^2 \qquad (3.9)$$

$$Ft = \rho\ Cd\ \bar{V}\ Vt \qquad (3.10)$$

As the relation between force and velocity is linear, it follows that the force is also gaussian, and coefficient of drag, Cd, is a function of the frequency f.

$$S_F(f) = (\rho\ \bar{V}\ Cd)^2\ S_v(f) \qquad (3.11)$$

$$\sigma_F = \rho\ \bar{V} \left[\int cd^2(f)\ S_v(f)\ df \right]^{1/2} \qquad (3\ 12)$$

substituting the expression for $\bar{F}$, in the above equation we get

$$\sigma_F = \bar{F} \left[\int \lambda^2(f) \quad \frac{S_v(f)}{(\bar{V})^2} \quad df \right]^{1/2} \qquad (3.13)$$

where $\lambda(f) = 2\ Cd(f)/Cd(0)$, and $Cd(0)$ is the drag coefficient in mean flow. Similarly one can express the displacement in terms of the force as $\bar{Y} = \bar{F}/\ k$, where k is the stiffness as defined by Eq (3.3).

The standard deviation of the displacement, σ_Y in terms of the mean displacement $\bar{Y}$ is given by

$$\sigma_Y = \bar{Y} \left[\int \lambda_m^2(f)\ \lambda^2(f)\ \{ S_V(f)/\ (\bar{V})^2 \}\ df \right]^{1/2} \qquad (3.14)$$

where $\lambda_m^2(f)$. is obtained by multiplying the complex frequency response function $|H(f)|^2$ by $4\pi^2 f^2$, and is called the *mechanical*

admittance. For determining the largest peak response, the probability density function is defined for the distribution of peaks for a zero mean, stationary and gaussian process $Y(t)$, with a standard deviation σ_Y as

$$q_Y(\eta) = \frac{\nu T}{\sigma_Y} \eta \, e^{-\eta^2/2\,\sigma_Y^2} \qquad (3.15)$$

where ν, is the average frequency of positive crossings and T is the period over which the record is averaged.

3.3 Wind Spectra for Alongwind Response

On the basis of averaging several wind measurements over terrains of different surface roughness, Davenport suggested an empirical relationship for the spectrum of horizontal gustiness in high winds as

$$\frac{f\, S_V(f)}{V_*^{\,2}} = 4 \, \frac{x^2}{(1 + x^2)^{4/3}} \qquad (3.16)$$

where, $x = 1200(f \,/\, V_1)$

V_* = Shear velocity defined in sec 3.

f = frequency

V_1 = Mean hourly wind speed at 10 m, in m/sec

The spectrum has been obtained by averaging different velocity spectra for different heights and as a result is independent of the height of structure. In 1974 Simiu had suggested a height dependant spectrum that could be used as a correct representation

of the entire spectra (both low and high frequency) as

$$\frac{f\,S_v(f)}{V_*^2} = \frac{200\,f}{(1 + 50\,f)^{5/3}} \qquad (3.17)$$

where, f = $fz/V(z)$ called the *Monin* or *similarity coordinate* This expression approximates very closely the spectrum in the high frequency range. Table 3.1, shows the comparison of the spectrum values of the foregoing spectra at 100 m height, and for a wind speed of 33 m/s at 10 m height. From the table it is clear that the height independent spectrum over estimates the longitudinal spectra of turbulence as much as 100 - 300 %.

Table 3.1 , Comparison of $fS_v(f)/V_*^2$ values, as per eq(3.16) and eq(3.17) , z = 102 m, V_1 = 33m/s

Frequency (f)	$f\,S_v(f)/V_*^2$	
	Eq (3.16)	**Eq (3.17)**
0.1	1.534	0.698
0.2	1.04	0.472
0.5	0.576	0.268
0.7	0.461	0.215
1.0	0.364	0.171
1.2	0.323	0.152
1.5	0.279	0.131
1.7	0.256	0.121

3.4 Cross-Spectra of Longitudinal Velocity Fluctuations

For characterization of the correlation between the velocity fluctuations at two different points in space, it is also necessary to have the *cross spectrum* of these two processes. The *cross spectrum* of two continuous records is a measure of the degree to which the two records are correlated and is defined by the expression

$$S^{cr}_{v1\ v2}(r,f) = S^{c}_{v1\ v2}(r,f) + i\ S^{q}_{v1\ v2}(r,f) \qquad (\ 3.18\ )$$

where r is the separation of two points *M1* and *M2*, and v_1 and v_2 are the respective records taken at the above two points. $S^{c}_{v1v2}(r,f)$ is the in-phase co-spectrum and $S^{q}_{v1v2}(r,f)$ is the quadrature spectrum. Generally the latter is small enough to be neglected, while the former is represented by the *Coherence function*, for which Davenport had suggested the following expression

$$\mathrm{Coh}(r,f) = e^{-\mathbf{f}} \qquad (\ 3.19\ )$$

$$\text{where } \mathbf{f} = \frac{2n\sqrt{C_y^{\,2}(\ y_1 - y_2)^2 + C_z^{\,2}(\ z_1 - z_2)^2}}{V(\ z_1) + V(\ z_2)}$$

In the foregoing expression y_1, y_2 and z_1, z_2 are the coordinates of the points *M1* and M2 respectively. The line passing through the two points is assumed to be perpendicular to the direction of

mean wind. C_y and C_z are the experimentally determined exponential decay coefficients.

3.4 Alongwind Response of Tower

The following assumptions have been made while arriving at an expression for the along wind response of a tower

(a) The mode shape has been assumed as $\{ 1 - \cos(\pi x/2h) \}$, where x is the dimension along the height of the tower and h is the height of the tower.

(b) Spectrum of horizontal gustiness as proposed by Simiu, and given earlier by eq(3.17) has been used.

(c) The along wind cross spectra of longitudinal velocity fluctuations have been taken as equal to unity, as for the tower there is no variation in the pressure coefficients between the windward and leeward side.

(d) The across wind cross spectra in the vertical direction as proposed by Davenport, and given earlier by eq(3.19) have been used.

(e) The contribution of the higher modes of vibration to the response has been ignored.

(f) All calculations are based on the mean hourly wind speeds.

A tower can be idealized as multi degree of freedom system(MDOF), with the masses lumped suitably at different heights. The equation of motion of a MDOF system can be written as

$$[M]\ \ddot{x}(t) + [C]\ \dot{x}(t) + [K]\ x(t) = P(t) \qquad (3.20)$$

Where [M], [C], and [K] are the mass, damping and stiffness

matrices of the tower.

Expanding $X(z,t)$ in normal coordinates, we get

$$X(z,t) = \sum_{i=1}^{n} x_i(z,t) = \sum_{i=1}^{n} \phi_i(z)\ q_i(t)$$

and substituting in Eq(3.20), and by mode superposition and orthogonality of mode shapes we get

$$M_n \ddot{q}_n(t) + C_n \dot{q}_n(t) + K_n q_n(t) = P_n(t) \qquad (3.21)$$

where $M_n = \phi_n^T [M]\ \phi_n$

$C_n = \phi_n^T [C]\ \phi_n$

$K_n = \phi_n^T [K]\ \phi_n$

$P_n = \phi_n^T\ P(t)$

The response of a SDOF damped system as governed by the above equation can be obtained using the Convolution or Duhamel integral

$$q(t) = (1/\ m\omega_d) \int_0^t p(\tau)\ e^{-\zeta\omega(t-\tau)}\ \sin \omega_d(t-\tau)\ d\tau \qquad (3.22)$$

The unit impulse response for a damped system is given by

$$h(t-\tau) = (1/\ m\omega_d)\ e^{-\zeta\omega(t-\tau)}\ \sin \omega_d(t-\tau) \qquad (3.23)$$

$$q(t) = \int p(\tau)\ h(t-\tau)\ d\tau$$

Response of a SDOF system in the frequency domain now can be written as

$$q(t) = (1/2\Pi) \int H(i\omega)\ c(i\omega)\ e^{-i\omega t}\ d\omega \qquad (3.24)$$

where ω is the variable frequency, $H(i\omega)$ is the complex frequency response function, and $c(i\omega)$ is the Fourier transform of forcing function $P(t)$.

$$P(t) = c(i\omega)\ e^{-i\omega t} \qquad (3.25)$$

$$q(t) = H(i\omega)\ c(i\omega)\ e^{-i\omega t}$$

Substituting in eq(3.2) the expression for $H(i\omega)$ is

$$|H(i\omega)|^2 = \frac{1}{k_n\{1 - \beta^2 + i\ 2\ \beta\ \zeta_n\}} \qquad (3.26)$$

where $k_n = M_n\ \omega_n^2$, the generalized stiffness of mode n, ζ_n is the modal damping, ω_n is the angular frequency of mode n, and β is the non dimensional parameter and is equal to ω/ω_n.

From the principles of random vibrations the basic relationship between the input and output spectra for a linear SDOF system is given by

$$S_0(f) = |H(f)|^2\ S_I(f) \qquad (3.27)$$

where $S_0(f)$ = The output spectrum

$S_I(f)$ = The input spectrum

$|H(f)|^2$ = Square of the modulus of structure frequency response function

The expression given in Eq(3.26) for the square of the modulus of the structural frequency response function when multiplied by M ω_n^2, gives a dimensionless coefficient called the mechanical admittance denoted by $|\lambda_m(f)|^2$.

$$|\lambda_m(f)|^2 = \left[\{ 1 - (f/f_n)^2\}^2 + \{ 4 \quad \delta^2 (f/f_n)^2\} \right]^{-1} \qquad (3.28)$$

where f = frequency

f_n = natural frequency of mode, n

δ = damping in the n th mode

Damping δ, is a sum of both the structural and the aerodynamic damping.

$$\delta = \delta_{str} + \delta_{aero} \qquad (3.29)$$

δ_{str} = 0.02, for steel structures

$$\delta_{aero} = \rho \sum \frac{C_f \; A_e(z) \; V(z) \; \phi^2(z)}{2 \, \omega_n^2 \, M_n}$$

The relationship between the mean deflection Y, and its standard deviation $\sigma(Y)$ can be given by the expression

$$\sigma(Y) = Y \left[\int |\lambda_m(f)|^2 \; |J_r(f)|^2 \; \{S_v(f) / V^2(z)\} \; df \right]^{1/2} \qquad (3.30)$$

where $|J_r(f)|^2$ = Joint acceptance for the r^{th} mode, and measures

the correlation between the spatial distribution of pressure across the tower.

$$|J_r(f)|^2 = \{ 1/ N_r^2 \} \int_0^h \int_0^h R(z,z',f) \; \phi_r(z) \; \phi_r(z) \; dz \; dz' \qquad (3.31)$$

where $R(z,z,f)$ is the vertical cross correlation coefficient, and N_r is equal to $\int_0^h \phi_r^2(z)\, dz$. Eq (3.30) can now be written as

$$\sigma(Y) = (Y/N_r)\left[\int|\lambda_m(f)|^2\left(e^{-7hf/\,V(z)} \int_0^h \phi_r^2(z)\; dz\right)\{S_v(f)/\, V^2(z)\}\; df\right]^{1/2}$$

$$(3.32)$$

3.6 Gust Response Factor

As defined in the foregoing sections the gust response factor can be expressed as

$$G = 1 + G\{\, \sigma(Y)/\, Y \,\} \qquad (3.33)$$

where *G*, is defined as the *gust factor* and is given by the expression

$$G = \sqrt{2 \ln \nu T} \qquad + \qquad 1/\sqrt{2 \ln \nu T} \qquad (3.34)$$

T = Time scale of turbulence, taken as 3600 seconds

$$\nu = response\ factor = \left[\frac{\int f^2\, S_y(f)\; df}{\int S_y(f)\; df}\right]^{1/2}$$

3.7 Comparison of Codal Provisions.

The procedure for calculating the gust response factors, in different codes of practice have all essentially been derived from the method for calculation of alongwind response, explained in section 3.3. The variations in the values obtained are because of the assumptions made. Table 3.2, shows the comparison of gust factor values for a 102 m tower, between the BS: 8100 and the procedure explained in the foregoing sections, for different terrain conditions as specified in the above mentioned standard. All the values are based on the total height of the structure, and hourly mean wind speed of 33 m/s.

Table 3.2, Comparison of Gust factor values, for 102 m square tower, and hourly mean wind speed = 33 m/s

Terrain roughness Coefficient (Z_o)	Power law index of variation (α)	Gust factor	
		Eq (4.21)	BS: 8100
0.003	0.125	2.04	1.62
0.01	0.140	2.18	1.81
0.03	0.165	2.34	2.01
0.10	0.190	2.62	2.38
0.30	0.230	2.87	2.81

The values obtained by the method outlined earlier are quite comparable to the values as per BS: 8100, in terrain conditions III to V.

BS: 8100, however has given two methods for calculation of the gust response factor. The first method calculates the gust factor at different points along the height of a tower. The simplified alternative calculates one value of the gust response factor for the entire tower. A comparison of the two methods outlined in the BS: 8100, for calculation of the gust response, shows that the simplified alternative is quite conservative by up to about 30 - 40 %.

IS: 875, unlike the British standard is a general code that deals with the loading provisions for all structures. In IS :875, section 7 deals with the dynamic effects. Herein the method of computing the alongwind response of structures has been explained. The code has introduced a coefficient, $\bar{K}_2$ called the hourly mean wind speed factor for different terrains and different heights, that converts the 3 second peak gust speed to mean hourly wind speed. Table 3.3 ,shows the comparison of the gust response factor values between the procedure outlined in the preceding sections and the provisions as per IS: 875. Since the method outlined in IS: 875, pertains to all kinds of structures, the values have been calculated based on the average width of 7 m, peak 3 second gust speed of 50 m/s. The terrain coefficient values are as explained earlier in Sec 2.3.4.

Table 3.3, Comparison of Gust factor values, for 102 m square tower, and hourly mean wind speed = 33 m/s

Terrain roughness Coefficient (Z_0)	Gust factor	
	Eq (4.21)	IS :875
0.002	1.96	1.77
0.02	2.18	2.02
0.2	2.51	2.20
2.0	3.05	2.56

The values of the gust factor as per IS: 875, are less than the computed values by about 10 - 20 %. This may be because of the fact that the provisions of the IS code cover primarily only buildings and closed structures and not lattice towers. The comparison has however been made to bring out the inadequacy of the code in its present form. Detailed comments have been made in Chapter 5, while discussing the shortcomings of the code. However it will be sufficient here to note that, for the dynamic analysis of lattice towers and masts, the present IS code should not be referred

ASCE: 7-93, has given an expression to calculate the gust factor, particularly for open framework or lattice structures This method for calculating the gust factor is being followed in the US for design of tall microwave towers. Table 3.5,gives the values of the gust factor calculated as per the above mentioned code, for a 102 m tower.

Table 3.5, Gust factor values as per ASCE: 7-93

Exposure Category	Gust Factor
A	1.284
B	1.204
C	1.050
D	0.984

The values of gust factor in the above table are based on a mean wind speed corresponding to the fastest mile wind speed. Having seen the methods adopted by different codes of practice for computing the gust response factor and also the analytical method of arriving at the alongwind response of a tower, it is important to see the effect of these values on the overall wind loading. Table 3.5, shows a comparison of the total wind load per panel for a 102 m tower calculated by the different methods, for flat open terrain (terrain category II as per IS: 875, terrain category III as per BS: 8100 and exposure category C, as per ASCE: 7-93)

Table 3.5, Comparison of overall wind load per panel

Panel	Wind load per panel (kN)		
	IS :875	BS :8100	ASCE :7-93
1	14.43	13.05	12.00
2	14.24	13.67	11.79
3	18.00	17.82	14.72
4	19.82	19.91	16.74
5	21.44	21.74	18.21
6	22.94	23.36	19.44
7	22.72	23.10	19.11
8	23.28	23.65	19.43
9	23.72	24.03	19.63
10	27.03	27.54	22.40
11	27.19	27.65	22.57
12	27.08	27.28	22.19
13	29.54	29.85	24.35
14	28.56	28.65	23.57
15	29.54	29.63	24.84
16	25.87	22.78	22.68
17	17.65	13.096	18.61

From Table 3.5, it is evident that the final wind load per panel is nearly the same irrespective of the code referred. The differences in the values are because of the differences in the factors of safety and wind profiles over the height adopted by the respective codes. While BS: 8100 has specified a value of 1.20, IS: 875 and ASCE: 7-93 have specified a value of 1.07, for the factor of safety.

CHAPTER IV

RESULTS

4.1 General

In the preceding chapters, the codal provisions regarding wind loading on structures had been examined. With the advent of digital computers analysis of a three dimensional space truss is no longer a problem. Today, there are many engineering software packages, that analyze space truss and space frames. As mentioned before the **SAP 80** package was used. For parametric studies, a number of towers need to be analyzed; for which preparing the input in the required format can be tedious. Hence a pre-processor program was developed in FORTRAN, which generates the required input for carrying out both static and dynamic analyses. Parametric studies have been carried out on 102 m towers of both square and triangular in-plan configuration, for different H/D ratios. The optimum member size for each configuration has been identified by comparing the stress ratios obtained with the safe allowable stress ratios.

4.2 Description of the Pre-processor package

4.2.1 Main Program

The pre-processor package comprises of a main program and six subroutines. The main program generates the co-ordinates of the member joints, the member connectivities for both type of towers. The program generates three types of bracing, namely the **'X'**,

'**XB**', and the '**K**' type of bracing. Only the leg members and the primary bracing members are generated, while allowance for the effects due to secondary bracing is made in all other calculations. The program generates dummy nodes that are required to define the orientation of the member axes. Fig4.1 is a flow chart that explains the working of the pre-processor package.

4.2.2 Sub-routines

MATPROP

This sub-routine calculates the member properties like moment of inertia (MI), area of cross section (A), weight per unit length (W). The input information required is the outer diameter and thickness of the member.

DEADLOAD

This sub-routine calculates the dead load (load condition, L=1), on the tower due to the weights of antenna, tower appurtenances, and secondary fitment items. The member weights are mentioned as the weight per unit length (W). The program automatically calculates the weight of the tower.

WINDLOAD

This sub-routine calculates the static wind load on the tower due to face and diagonal wind (load condition, L=2 and L=3 respectively).The program requires the basic wind speed as input and generates the variation of wind speed with height in accordance with the provisions of IS :875, for terrain category II. It also calculates the solidity ratios (ϕ), and the panel wise force coefficients (**FC**), all as per the above mentioned code

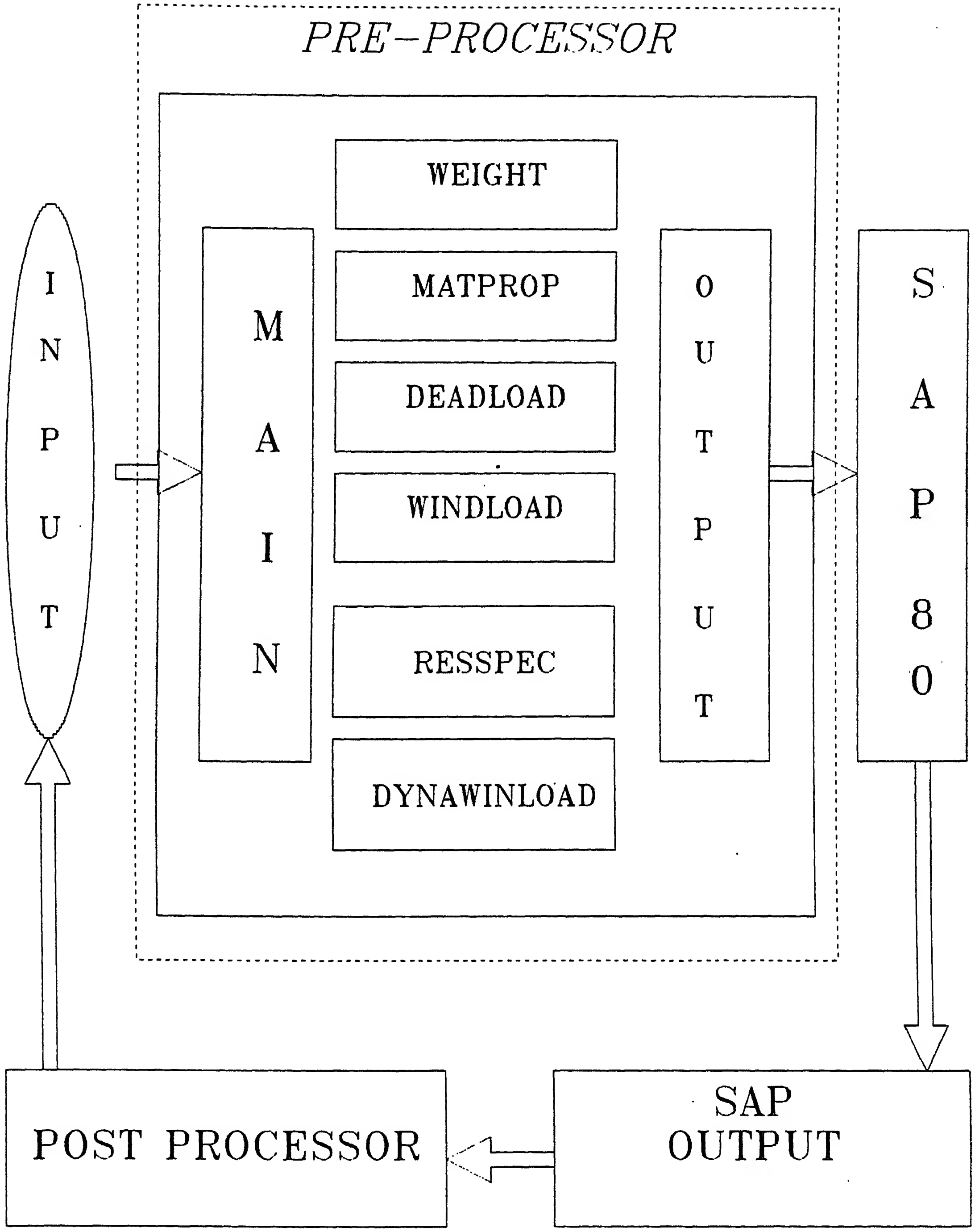


Fig 4.1 Flow chart showing the working of the pre-processor

for rounded members and sub-critical flow regimes. The force calculated at the center of the panel is distributed to the joints depending on the type of bracing. The basic wind speed that is to be input the peak 3 second gust speed as mentioned in the code.

DYNAWINLOAD

This sub-routine generates the wind load due to the gusts(load condition, L=4). The basic wind speed is converted to the mean hourly wind speed and the variation of wind speed with height is done in accordance with the logarithmic law as mentioned earlier in Chapter 2. For calculation of the alongwind response and gust response factor, the procedure outlined in the previous chapter has been followed. The terrain roughness coefficient values of 0.002, 0.02, 0.2, 2 for the terrain categories I to IV have been adopted.

WEIGHT

In this sub-routine, the weight of each panel is calculated and the masses suitably lumped at the middle of each panel.

RESSPEC

This sub-routine generates the response spectrum for 2% damping as per IS: 1893. The importance factor, soil foundation factor, performance factor have all been taken as per the above mentioned code. The response spectra is generated depending upon the seismic zone where the tower is to be designed.

4.2.3 Input Data

The information that is to be input is as under

(a) Type of analysis (0/1,truss or frame), Seismic zone (I to V)

(b) Number of sides in plan (nsp=3 or 4), total height of tower in meters (toth), top width (tw), bottom width (bw)

(c) Number of prismatic panels (npp), number of non-prismatic panels with **X** bracing (nnpx), number of non-prismatic panels with **K** bracing (nnpk).

(d) Number of different material types (nmat), member number (mid) and the outer diameter (od1) and thickness (t) of each material type.

(e) Panel-wise member number of leg and bracing.

(f) Peak 3 second gust speed (V_b), as per IS: 875.

4.3 Results

Before carrying out the analyses, the influence of the choice of analysis ie, space frame or space truss on the results needs to be examined. Fig 4.2, shows the comparison of the displacement

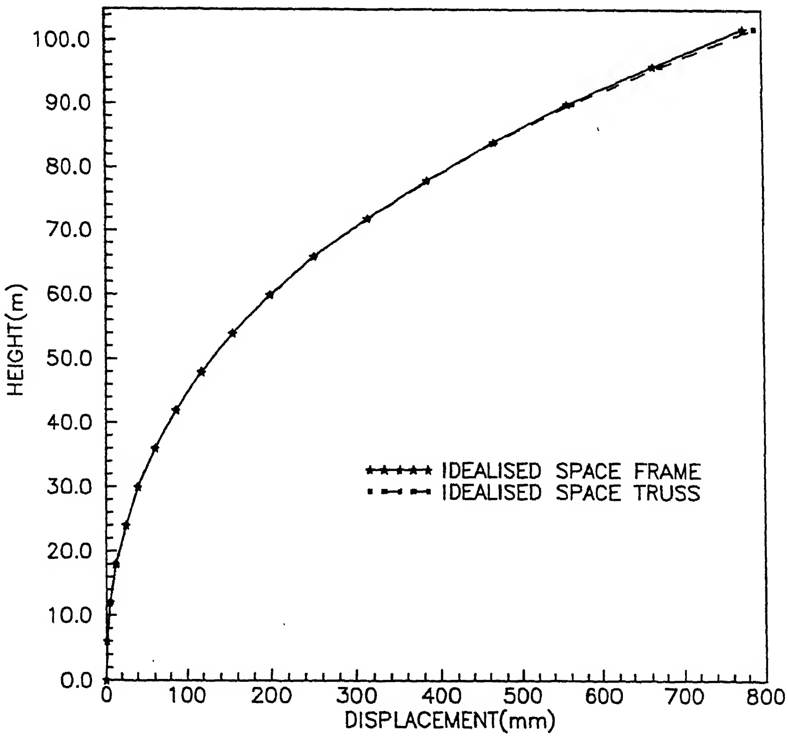


Fig 4.2 Comparison of displacement for different analyses.

of a 102 m square tower by the two analyses. As seen from the figure the displacement profile in either of the analysis is quite similar, because the lateral loads have been distributed only at the joints, while actually in the frame analysis the loads are to be uniformly distributed along the member length. In all the following comparisons being made, the tower has been idealized as a space truss.

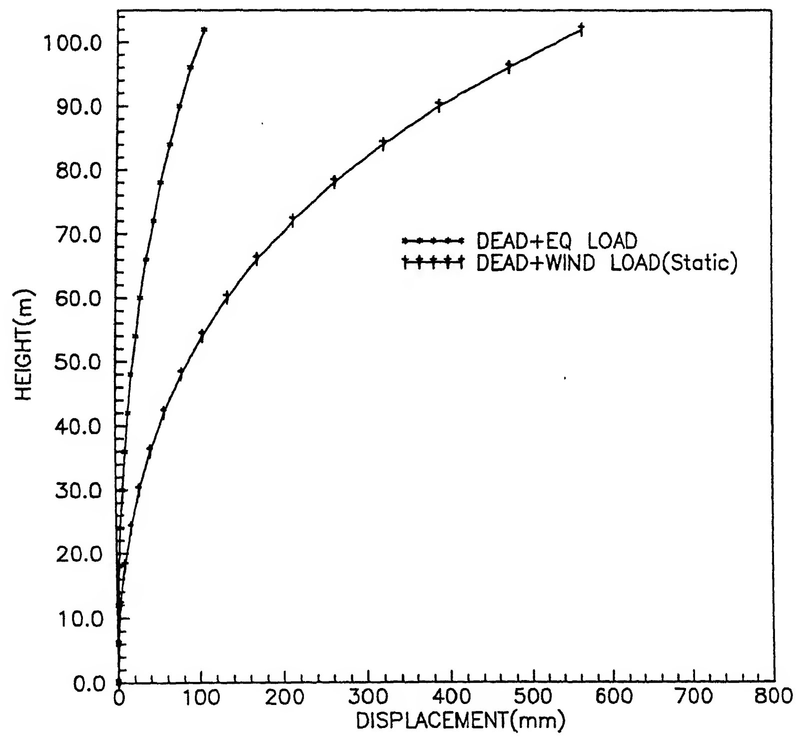


Fig 4.3 Comparison of displacement for worst load combinations.

Fig 4.3, shows the comparison of displacements for a square tower, H/D ratio equal to 4, considering the combination of dead and wind loads and the combination of dead and earthquake loads. It is seen that for a tall and flexible structure the critical load combination is the dead and dynamic wind load and not the

earthquake load. However for the design of some bracing members the earthquake load could play an important part.

For the purpose of analysis an initial section was assumed and the analysis carried out. The stresses were then compared with the safe allowable stresses, and the sections were changed if the values were exceeded. Fig 4.4, shows the initial section profile for the leg members that was assumed for a 102 m square and triangular tower. As can be seen a greater diameter section has been assumed in case of a triangular tower for the reason that the force to be resisted by the leg member is greater.

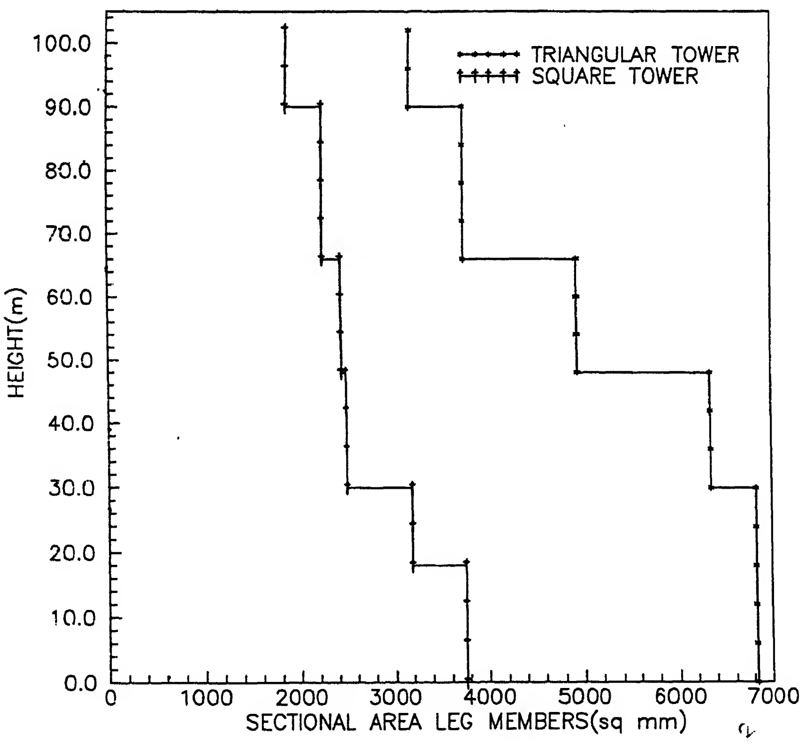


Fig 4.4 Initial assumed section profile for leg members of 102 m tower.

Based on the initial section profile assumed, analysis of

uare towers with different H/D ratios was made. Fig 4.5, shows he displacement profile of the towers. From this figure it is that the tower with H/D=7 (base width of 10.3 m proximately) has a tip displacement of nearly 1.35 m, while the wer with H/D=4 (base width of 18.03 m approximately) has a rresponding tip displacement of about 0.5 m. Keeping in mind limitation of tip displacement mentioned earlier in Chapter only the towers with H/D ratios of 4 and 5 are safe while in ne other two towers the section profile need to be changed.

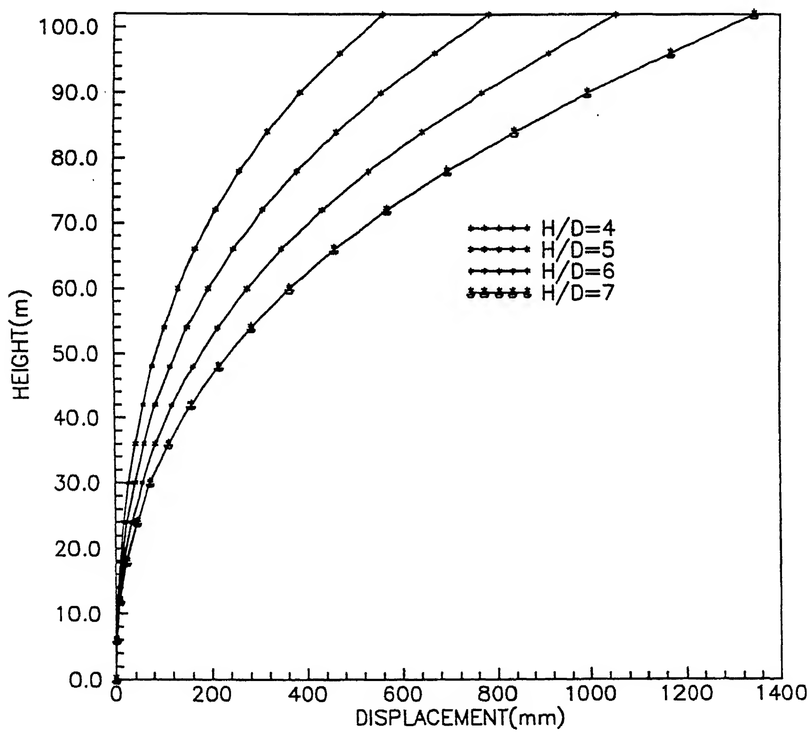


Fig 4.5 Initial displacement profile for 102 m, square tower, for different H/D ratios.

Fig 4.6, shows the force profile of the leg members for a wer with H/D = 5. It is seen that the values of the tensile and

ompressive forces, are nearly the same along the height of the
ower.

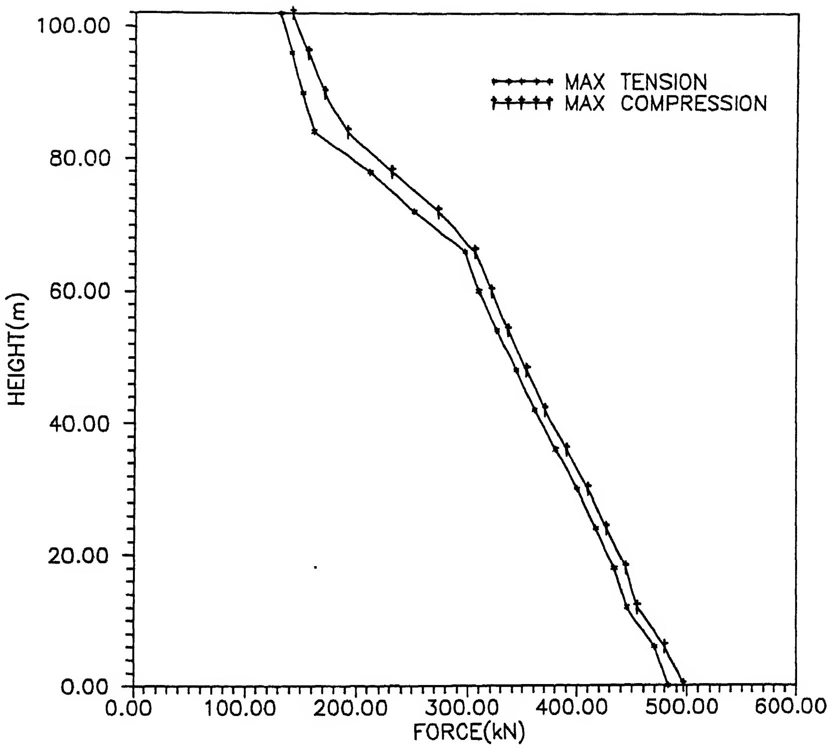


Fig 4.6 Force profile in leg members, 102 m square towers.

Having changed the initial section assumed, Fig 4.7, shows
e new displacement profile for the towers. It is seen that the
ximum tip displacement in the tower with H/D = 7, has now been to
duced to about 0.9 m. Table 4.1 shows the final stress ratios
the leg members of all the towers compared above.

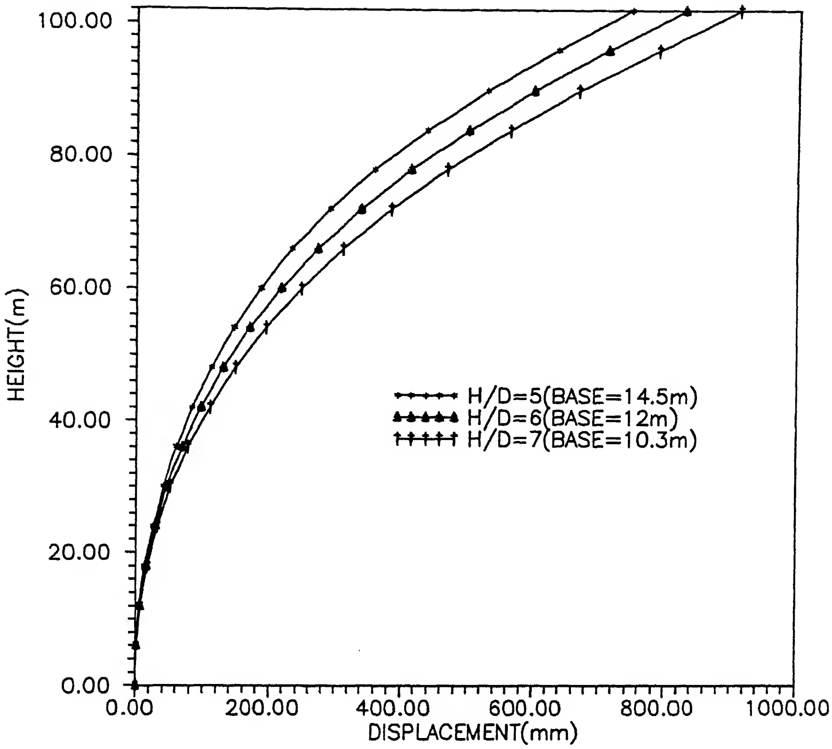


Fig 4.7 Modified displacement profile, 102 m square tower, for different H/D ratios.

Table 4.1, Final stress ratios for square towers in leg members.

Panel	H/D = 4		H/D = 5		H/D = 6		H/D = 7	
	T	C	T	C	T	C	T	C
1 - 2	0.50	0.64	0.50	0.64	0.50	0.64	0.50	0.64
3 - 6	0.68	0.82	0.79	0.98	0.84	1.03	0.79	0.97
7 - 9	0.70	0.88	0.85	1.03	0.84	1.06	0.83	1.03
10 - 12	0.77	0.96	0.81	0.99	0.81	1.01	0.84	1.05
13 - 14	0.72	0.90	0.77	0.96	0.80	0.96	0.85	1.05
15 - 17	0.69	0.87	0.86	1.06	0.81	0.96	0.76	0.93

In the above table T stands for tension and C denotes compression.

From the table it can be seen that while most of the values are less than 1, some values are marginally over 1. The reason for accepting such members as safe, is that the stresses in those members whose values exceed the safe allowable values can be reduced by the introduction of secondary members.

An important point to note in optimization is the overall weight of structure being optimized. Table 4.2, shows the comparison of the weights of square towers. The weight of a tower with narrow base is nearly 10% greater than the weight of a tower with broader base.

The values in the table are only of the leg and primary members, and do not include the weights of the secondary members, appurtenances and connections.

Table 4.2, Comparison of weights, square towers

Base width (m)	Initial weight (kN)	Final weight (kN)
10.30	225	297
12.00	238	289
14.50	250	282
18.03	277	277

Similar parametric study has been carried out on triangular towers. Fig 4.8, shows the the initial and final displacement profile for a

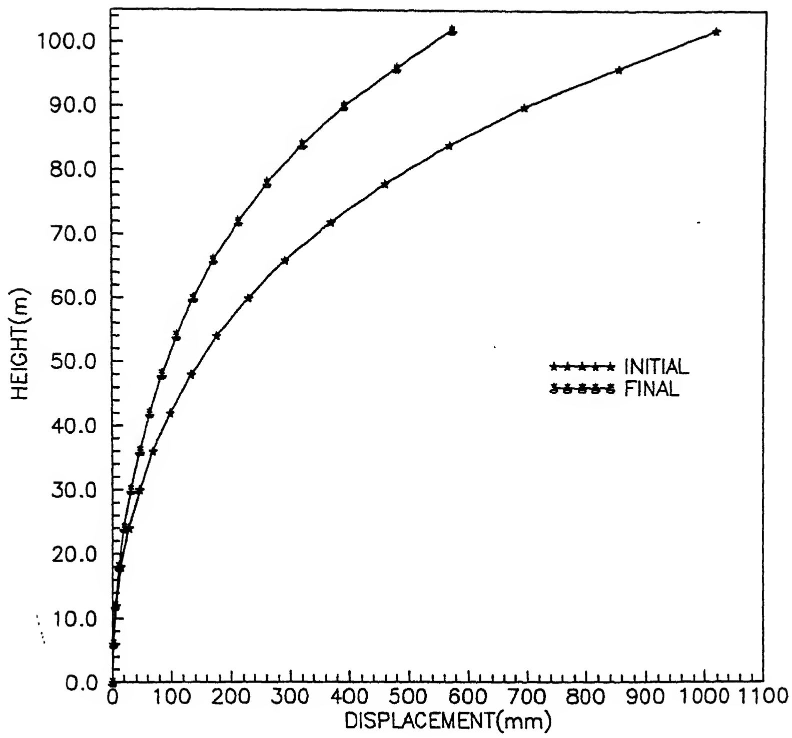


Fig 4.8 Initial/Final displacement profile, 102 m triangular towers.

102 m triangular tower with H/D ratio equal to 5. Fig 4.9, shows the force profile in the leg members. Here it can be seen that the maximum compressive force is nearly twice the maximum tensile force as in the *face wind condition* there is only one leg that resists the force whereas for the square tower, there are two legs resisting the same force

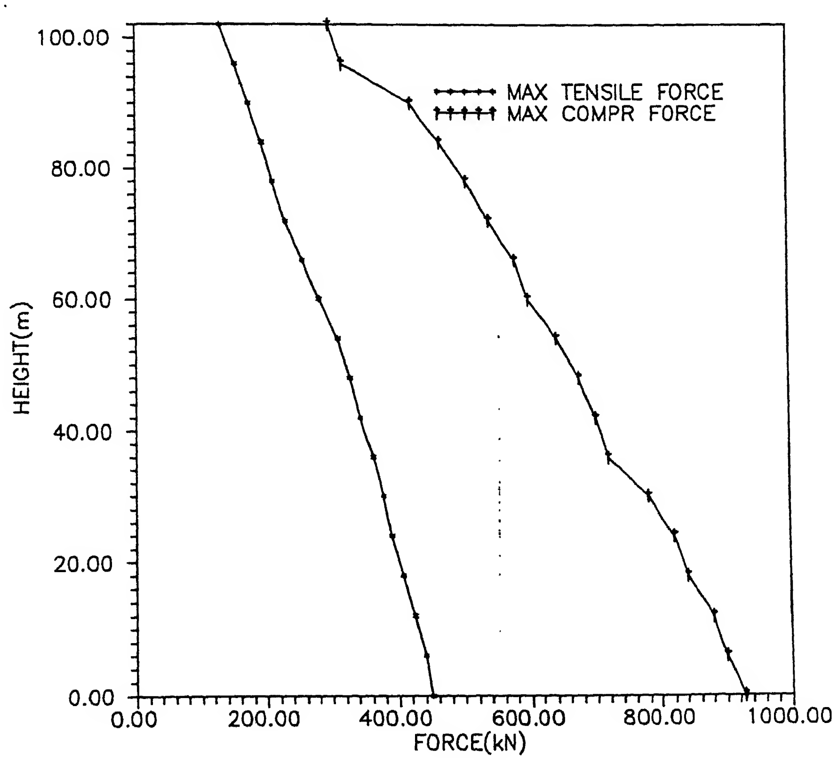


Fig 4.9 Force profile in leg members, 102 m

CHAPTER V

CONCLUSIONS

5.1 Conclusion

Having studied the codal provisions of the three countries in detail with special emphasis to wind loading, the following can be concluded:

(a) The reference wind speeds mentioned in the codes vary as their averaging times are different. Therefore before referring to any code, this point needs to be noted.

(b) While the wind pressure is dependant on the wind speeds and their respective averaging times, the force coefficient depends mainly on the solidity ratio of the panel, orientation of the tower with respect to the direction of mean wind and plan shape of the tower and the shape of the component members. So for the same design wind pressure and tower properties, it is natural to anticipate identical values for the force coefficients irrespective of the code of practice (with marginal allowance for variation). But the variation observed in the values of the overall force coefficient is around 10-20%.

(c) In tall towers the difference in the solidity ratios between the top and bottom panels is about 30-40%, depending upon the stiffness of the structure. Therefore taking an average value of the solidity ratio and thereby computing the overall force coefficient results in values of wind

loads that are conservative in the top few panels, but under estimate the wind loads by about 10-15% in the panels close to the ground. It is therefore recommended that while calculating the force coefficients cognizance be taken of the panel-wise solidity ratios for better results.

(d) The response calculated by using the height independent spectra for longitudinal velocity fluctuations is approximately 30% higher than the response calculated by the height dependant spectra.

(e) The gust response factor varies as the terrain roughness coefficient changes. It is seen that cities have the greatest gust factors while flat and open terrain have correspondingly low values. Mean wind speeds are however lower in the cities as compared with flat and open terrains.

(f) Values of gust factor as per BS: 8100, compare well with the analytical method explained in Chapter 3. It is however incorrect in comparing the values of the gust factor obtained from different codes, unless the averaging times for the reference wind speeds are the same.

(g) Although the methods for computing the dynamic alongwind response are different in the three codes, the final values of the panel-wise wind load are nearly the same. Therefore till a comprehensive and dedicated code is formulated for lattice towers in India, one may refer to BS: 8100 after suitably adopting a value for the factor of safety.

(h) For small towers up to 100 m in height, it is recommended that tubular sections be used to achieve a

considerable reduction in the weight of the structure. It is seen that a 100 m tower made up of angle section weighs about 50-60% more than a similar tower made from circular sections.

(j) It is observed that as the base width of the tower increases, the structure becomes more stable and the maximum deflection at the top reduces.

(k) It was seen specially in the case of triangular towers, that in some cases the design required the use of tubular sections that were larger than the maximum specified sections as per IS: 1161, thus placing a restriction on the height of the tower that could be designed.

(l) The maximum height for which the circular sections could be safely used, for a H/D ratio of 5, in open terrain was about 110 m and 140 m for triangular and square in-plan configurations respectively.

(m) The other obvious advantage in using circular section is that the radius of gyration for a given area of cross section is greater than an angle section. Moreover with pipe sections for leg members, any in-plan configuration of the tower can be provided.

(n) However use of tubular sections is not without its share of problems. Connections both bolted and welded, need a lot of precise detailing. Fabrication requires a very high standard of workmanship and skill.

5.2 Critical View of IS: 875-1987, Part III

The IS: 875 cannot be faulted in its present form as it covers the wind loading provisions for a wide range of structures. It is specially tailor made for design of buildings and other closed structures, that do not require a detailed study of their dynamic behaviour. The code has identified its inadequacies when it comes to specialist structures, and has cautioned the designer to carry out investigations with the aid of analytical methods. Notwithstanding the above some areas that are ambiguous and require deliberation are enumerated as under:

(a) While the provisions relating to force coefficients compare well with other current codes of practice on lattice towers, it is felt that specific values for force coefficients for tower appurtenances need to be mentioned.

(b) An approximate method for computing the natural frequency of vibration of tower structure needs to be included. In this regard it is suggested that idealizing the tower as a cantilever, fixed at the base, and of width equal to the average width of the tower, is one method. The other method is to idealize the tower as a two dimensional framed structure and compute the natural frequency of vibration.

(c) The code must prepare a detailed method of computing the alongwind response by the gust factor method only for lattice structures on the lines of BS: 8100. The code must also specify the parameters to be adopted by designers if they wish to carry out approximations by analytical methods as an appendix to the code.

5.3 Scope for Future Work

The present work does not include any optimization technique to design the tower structure. It is felt that economy can be achieved by optimizing the structure using Genetic Algorithm and other new optimization techniques. Further design and modeling of connections is another aspect that needs to be looked into along with design of foundations for towers.

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